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6TH APRIL 2009 L'AQUILA EARTHQUAKE, ITALY: REINFORCED CONCRETE BUILDING PERFORMANCE

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ABSTRACT

On 6th April 2009 an earthquake of magnitude $M_w = 6.3$ occurred in the Abruzzo region; the epicentre was very close to the city of L'Aquila (about 6 km away). The event produced casualties and damage to buildings, lifelines and other infrastructures. An analysis of the main damage that reinforced concrete (RC) structures showed after the event is presented in this study. In order to isolate the main causes of structural and non-structural damage, the seismological characteristics of the event are examined, followed by an analysis of the existing RC building stock in the area. The latter issue came under scrutiny after the release of official data about structural types and times of construction, combined with a detailed review of the most important seismic codes in force in the last 100 years in Italy. Comparison of the current design provisions of the Italian and European codes with previous standards allows the main weaknesses of the existing building stock to be determined. Damage to structural and non-structural elements are finally analyzed thanks to photographic material collected in the first week after the event; the main causes of damage are then inferred.

KEYWORDS: L'Aquila, RC building, damage survey, old seismic design, brittle failure, infill.

1. INTRODUCTION

Field campaigns represent an important tool for design and retrofitting practice and can be considered the main basis for improvements in codes and local building practice. Examples of in-field campaigns after strong earthquakes ([Decanini et al. 2004](#); [Hosseini 2005](#); [Loh et al. 2003](#); [Rossetto and Peiris 2009](#)) also aim to identify the main structural causes of damage as well as the main characteristics of strong motion.

In this study, a photographic documentation that was collected in the first days after the L'Aquila earthquake mainshock is provided. Damage and structural performance of RC buildings are described and analyzed, and attempts are made to identify the main weaknesses of the building stock. The principal causes of structural and non-structural damage are determined by a three-step process.

The first step involves a data overview aimed at evaluating the impact of RC structures on the whole building stock and a detailed review of design codes in force at the time of construction of the buildings in the area. The second step involves analysis of the earthquake's main characteristics; the main aim of this step is comparison with seismic demand provided for by the codes. The third step involves analysis of the photographic documentation in the light of the previous steps, with a view to determining structural weaknesses that can be targeted as the main causes of global and local collapse and non-structural damage.

2. REINFORCED CONCRETE BUILDINGS IN L'AQUILA

The layout of the ancient city of L'Aquila is characterized by two major streets crossing at right-angles at a place called "Quattro Cantoni". The historical centre ([Figure 1](#)) is situated on a raised hill overlooking the surrounding area. It is encircled by medieval walls, which still stand almost completely undamaged.

The first structures beyond the ancient perimeter were the sports facilities in Viale Gran Sasso, built in the 1930s. However, urbanization accelerated after World War II, especially during the 1960s and 1970s, following the opening of the highway to Rome. In 1965 and 1975 two urban plans were enacted. Expansion mainly occurred in on the north-western side of the city, leading in a few years to the complete saturation of the urban area bounded by the same highway ([Figure 1](#)). Later expansion radiated out from the historical centre on all sides, except to the south-west, the course of the Aterno river ([Figure 1](#)).

The current urban structure of L'Aquila is that of an historical centre surrounded by densely-packed suburbs, comprising the quarters of Pettino, Santa Barbara and Torrone, and less densely populated areas in the North-Western, including Coppito, Sant'Antonio and Torretta quarters. The remaining area within the city's administrative boundaries includes several villages in the surrounding area.



Figure 1. L'Aquila: historical centre (black line), the Aterno river (blue line) and the highway (yellow line) (Google Earth©)

According to data from the Italian National Institute of Statistics (*Istituto Nazionale di Statistica*, ISTAT), see [Figure 2](#), collected in 2001, which represent the most recent official source for information about the building stock in Italy, and hence in the L'Aquila area, 24% of buildings are reinforced concrete structures, 68% masonry structures and 8% structures of unspecified type ([Figure 2b](#)). Data that identify age of construction of the buildings ([Figure 2a](#)) indicate that 55% of the entire stock was built after 1945. A low overall incidence of RC structures shows that after 1945 new masonry structures were still being built and that the number of RC structures increased gradually over time. Based on the distribution of number of storeys ([Figure 2c](#)), only 5% of the buildings have more than three storeys. Assuming that all buildings with more than three storeys are RC structures, it may still be inferred that the vast majority of L'Aquila RC buildings are no more than three storeys tall.

If the interstorey height is considered to vary between 3.0 and 3.5 meters, approximate formulation provided by the Eurocode ([CEN, 2003](#)) for RC frame structures gives for three-storey buildings a fundamental period of 0.4 seconds.

Working hypotheses assumed in this section, which were useful in defining points of comparison between codes, are mostly confirmed by representative samples of buildings employed in other field campaigns carried out for the same earthquake (e.g. [Liel and Lynch, 2009](#)).

According to previous observations, in the following section 3.1, when comparing different Italian Code spectral shapes, the comparison is focused on period values that

range from 0 to 0.4 seconds. The latter assumption allows a comparison between constant acceleration branches of the spectra. A general review of design prescriptions in recent decades is employed to identify the main weaknesses of the building stock and to finally compare seismic demand given by codes with the demand of the earthquake event considered in this study.

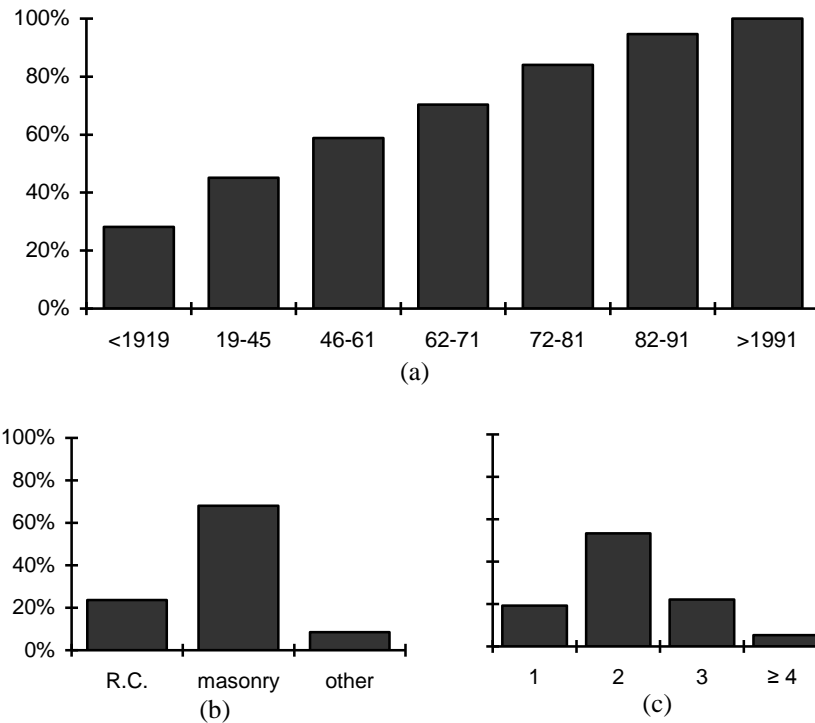


Figure 2. 2001 census ISTAT data for L'Aquila: (a) age of construction, (b) building type, (c) number of storeys

2.1 Seismic design criteria

L'Aquila and its neighbourhood were first legally recognized as a seismic zone in 1915. A specific law ([RDL 573, 1915](#)) was passed after the catastrophic seismic event occurring in January 1915 that struck Abruzzo (the Marsica earthquake) killing more than 29,000 people.

In 1927 ([RDL 431, 1927](#)) a more detailed seismic classification was introduced; this classification divided the Italian seismic area into two different categories; L'Aquila belonged to the less restricted one (second category). Seismic provisions were simply achieved by limiting the number of storeys: for the second category zone the 1927 law allowed construction of three-storey buildings, and for specific situations even four-storey buildings were accepted. Horizontal forces were equal to 1/10 of the storey

weight for structures up to 15 meters tall or 1/8 for structures higher than this limit. This code gave specific prescriptions for RC structures including prescriptions for dimensions of beam and column sections and the least amount of steel reinforcement required.

In the following years, between 1930 and 1937, three seismic codes were enacted ([RDL 682, 1930](#); [RDL 640, 1935](#); [RDL 2105, 1937](#)) and their main concern was evaluation of seismic forces. For seismic vertical action the additional load was determined as equal to 1/3 of the structural weight, thus reducing the amount applied in the 1915 regulations. In second category zones the ratio between seismic horizontal forces and vertical forces due to gravity loads (base shear coefficient) was initially 0.05 ([RDL 640, 1935](#)) and then 0.07 ([RDL 2105, 1937](#)). These prescriptions were confirmed in 1962 ([Legge 1684, 1962](#)) with less restrictive limits about building height and setting the maximum number of storeys at seven. Under this law further areas of the country were classified as seismic.

In 1975 ([DM 3/3/1975](#)) a fundamental innovation was introduced into the analytical procedure: for the first time within Italian regulations the dynamic properties of the structures were considered. Starting from this year, seismic action could be determined by means of static and dynamic analyses. In the static analysis, the resultant of lateral force distribution applied to the building was given by [Eqn. 1](#), where W is the total weight of the structural masses; R the response coefficient, assumed as a function of the fundamental period of the structure; coefficient C represents the seismic action ([Eqn. 2](#)), and is defined by means of S , the seismic intensity parameter. Coefficients ε and β respectively express soil compressibility ($\varepsilon=1.00-1.30$) and the possible presence of structural walls ($\beta=1.00-1.20$).

$$F_h = C \cdot R \cdot \varepsilon \cdot \beta \cdot W \quad (1)$$

$$C = \frac{S - 2}{100} \quad (2)$$

$$F_h = 0.07 \cdot W \quad (3)$$

For second category zones, S was assumed equal to 9. If coefficients ε and β were considered equal to 1, respectively corresponding to stiff soil and absence of structural

walls, a structure, whose fundamental period was lower than 0.8 seconds, was characterized by a base shear coefficient (F_h/W) of 0.07 and it was the same as that adopted up to 1975 (see [Eqn. 3](#)).

The coefficient given by the product of C and R should be interpreted as a design inelastic acceleration demand: it took into account dynamic properties of the structure and a strength reduction factor evaluated upon dissipative capacity of the structure. Nevertheless, lateral forces applied to the building were proportional to the height of the slab at each storey determined from the foundation level, assuming a linear distribution with an “inverted triangular” shape that is more suitable to represent the actual dynamic behaviour of the structure, compared with previous code prescriptions.

New generation codes explicitly express this kind of dissipative capacity of the structures; Eurocode 8 ([CEN, 2003](#)) does it by means of the “behaviour factor”. According to the Eurocode 8 definition, the behaviour factor q is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5% viscous damping to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure. The values of q , which also account for the influence of the viscous damping being different from 5%, are given for various materials and structural systems according to the relevant ductility classes.

Regarding seismic input, even if different new seismic design codes were approved ([DM 24/1/1986](#); [DM 16/1/1996](#)), no changes have been introduced regarding this aspect since 1975. On the other hand, in this period, the Limit State method was introduced and, for Ultimate Limit State assessment, design acceleration was supposed to be increased by a factor of 1.50, thus obtaining an acceleration of $(1.50 \times 0.07g)$, equal to 0.105g.

Furthermore, it is worth noting that the first prescriptions close to the *performance-based seismic design* approach, such as the attainment of both proper local and global ductility capacity, were provided in 1997 with an explanatory document attached to the 1996 code ([Circ. M.LL.PP. 65, 1997](#)). In this document there were limits for longitudinal and transversal reinforcement of beams and columns with specific prescription in the end zone of each structural element (critical region). In the 1997 document, additional prescriptions were provided regarding proper anchorage of bars,

and it was prescribed to lengthen longitudinal and transversal reinforcement of the column in beam-column joints. The latter prescription regarding beam-column joints was aimed at giving a proper local ductility to the element. Although this document ([Circ. M.LL.PP. 65, 1997](#)) represented an important step towards *performance-based design* criteria in Italy, lack in prescriptions about regularity criteria in plan and elevation is still recognizable and these criteria remained qualitative, without any specific quantitative definition to help in regularity classification.

In the 2003 seismic code ([OPCM 3274, 2003](#)) and its following modifications ([OPCM 3431, 2005](#)) an innovative seismic input definition was introduced, representing the first real upgrade towards the Eurocode 8 approach. In this document an elastic spectrum was provided with a defined shape in which the only value to be changed, considered a function of the seismic zone, was the anchorage Peak Ground Acceleration (PGA) on stiff soil type (ground type A). L'Aquila belonged to the second category and the PGA value on ground type A was 0.25g. This spectrum was to be amplified considering site specific characteristics, taking into account other ground types and topographic conditions. The elastic acceleration spectrum was to be reduced by the q factor value depending on the specific structural type, thus obtaining a design acceleration spectrum. This document explicitly introduced in Italy the *strength hierarchy* concept, ensuring the development of inelastic deformations in the highest possible number of ductile elements and not in elements with lower rotational capacity (that is, in beams and not in columns, due to the different axial load), but also providing over-strength to brittle failure mechanisms with respect to ductile ones. Furthermore, proper quantitative definition of regularity criteria in plan and in elevation was introduced, fixing maximum variation of mass, stiffness and strength over building height.

The most recent Italian code ([DM 14/1/2008](#)) defines maximum acceleration expected at the site no longer with division in terms of seismic zones but as a function of geographic coordinates of the site. For L'Aquila (latitude 42.38; longitude 13.35), the PGA is 0.261 g on ground type A for a 10% probability of exceedance in 50 years. In the case of L'Aquila, 2003 and 2008 PGA values are very close to each other.

2.2 Non-structural elements and damage limitation criteria

The 1975 and 1986 codes ([DM 3/3/1975](#); [DM 24/1/1986](#)) laid down the first stipulations concerning structural maximum allowable deformation under seismic loads, but verification was not mandatory unless specific requirements in terms of interstorey displacement limitation were necessary to preserve functionality of connection or restraint elements.

In the 1996 code non-structural damage limitation was first introduced. Assuming η_p and η_d are the elastic interstorey displacement demand respectively produced by earthquake loads and other loads, total interstorey displacement demand η_t was evaluated according to [Eqn. 4](#), where λ was considered a function of the importance factor and of the specific use of the building; it was given equal to 2, 3 and 4 for importance coefficient (I) respectively equal to 1.0, 1.2 and 1.4. When the Limit State method was employed in verification, a χ value of 1.5 was assumed.

$$\eta_t = (\eta_p \pm \lambda \cdot \eta_d) / \chi \quad (4)$$

The interstorey displacement demand evaluated according to [Eqn. 4](#) had to be lower than prescribed interstorey capacity in order to ensure that no expulsion of internal or external infill panels took place. Interstorey displacement capacity was limited to 0.002 h, where h was the interstorey height, if infill panels were brittle and firmly connected to the structure, or to 0.004 h if infill panels did not interact with structure deformation.

The 1997 explanatory document to the 1996 code ([Circ. M.LL.PP. 65, 1997](#)) further specified the evaluation of frame-infills interaction; in specific conditions, such as an effective connection between panel and frame, the contribution of nonstructural elements could be taken into account in structural analysis. Deformability of the composite frame-infills system could be determined assuming an equivalent strut model to account for infill presence, calibrated according to infill mechanical characteristics.

Infill verification took into account different collapse mechanisms (horizontal sliding failure, diagonal cracking failure and corner crushing failure) and axial load variation in columns due to interaction with infills had to be considered.

According to the 2003 seismic code ([OPCM 3274, 2003](#)) a damage limitation spectrum was defined dividing the elastic spectrum by a 2.5 reduction factor. Interstorey drift ratio (IDR), the ratio between interstorey displacement (evaluated by assuming the damage limitation spectrum as seismic input) and interstorey height, was compared with 0.005 when infills were firmly connected to the concrete frame or with 0.01 when infills did not suffer damage due to their deformability or to the nature of the connection with the surrounding RC frame. A comparison between 1996 and 2003 damage limitation requirements makes it evident that a higher IDR capacity was assumed but a more severe seismic input was considered.

Non-structural elements required additional and specific verification, which in the case of infill panels concerned possible out-of-plane failure mechanisms. This kind of verification could be ignored if specific design procedures aimed at avoiding brittle collapse or out-of-plane failure were adopted. Furthermore, a very irregular distribution of the infill panels in plan or in elevation (not quantitatively defined) was penalized by the code, assuming a higher value of seismic action, in order to account for possible damage concentration due to irregularity. Possible local interaction mechanisms in partially infilled frame bays were considered by specific and additional prescriptions on the amount of reinforcement and shear demand in columns adjacent to the infill panels.

The current seismic code in Italy ([DM 14/1/2008](#)) prescribes a specific Damage Limitation spectrum evaluated according to a hazard analysis corresponding to 63% exceedance probability in the considered life cycle of the building (50 years for ordinary buildings). Damage Limitation capacity limits are the same as those provided by the 2003 code and prescriptions about irregular distribution of infills are unchanged.

3. EARTHQUAKE CHARACTERISTICS

On 6th April 2009 an earthquake of magnitude $M_w = 6.3$ struck the Abruzzo region; the epicentre was only about 6 km from the city of L'Aquila. The event resulted in casualties and damage to buildings, lifelines and other infrastructures.

This was the third largest earthquake recorded by strong-motion instruments since 1972, after the 1976 Friuli ($M_w = 6.4$) and the 1980 Irpinia ($M_w = 6.9$) events. The event was generated by a normal fault, the epicentre coordinates being latitude 42.334 and longitude 13.334. The area has been hit by destructive earthquakes in the past, with

events being documented since 1300 BC (Stucchi et al. 2007). The three strongest earthquakes occurred respectively in 1349 (epicentral intensity $I_0=IX-X$ MCS), 1461 ($I_0=X$) and 1703 ($I_0=X$) (Ameri et al. 2009).

The L'Aquila earthquake mainshock was registered by 55 stations of the National Accelerometric Network (*Rete Accelerometrica Nazionale*, RAN), the stations being located from 4.8 to 278 km from the epicentre. The closest stations both to the epicentre and to the centre of L'Aquila are located on the fault trace (AQA, Aqv, AqG and AqK). The maximum PGA registered was 613.8 cm/s^2 on the East-West component of station Aqv, whose soil type was classified according to cross-hole test results as type B (Chioccarelli et al. 2009; Ameri et al. 2009).

Elastic response spectra of the four closest stations (Figure 3) are evaluated after baseline correction and filtering on the registered signals (Chioccarelli et al. 2009); both horizontal components (Figure 3a) and vertical components (Figure 3b) are reported. Vertical components of the registered signals showed quite strong spectral acceleration values especially in the high frequency range.

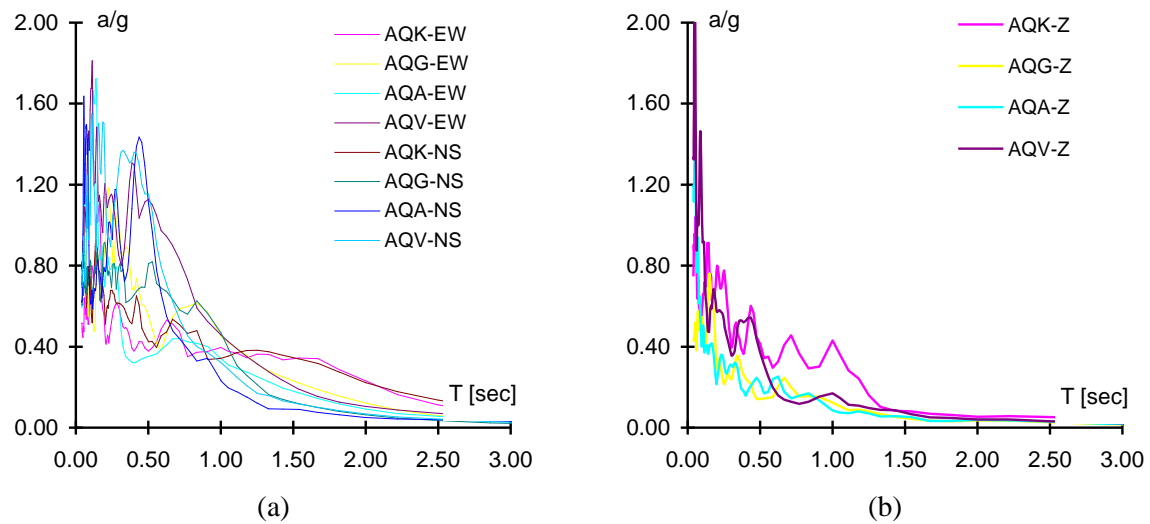


Figure 3. Elastic response spectra of horizontal (a) and vertical (b) components of L'Aquila mainshock registered signals (stations AQA, Aqv, AqG and AqK)

A reliable comparison with spectral results reported in Figure 3 is provided by Figure 4, where hazard maps for the L'Aquila area are reported in terms of PGA (see Figure 4a) and in terms of spectral acceleration for a 0.4-second period, $S_a(T=0.4)$, with 10% of

exceedance probability in 50 years on stiff soil (official Italian hazard data available at <http://essel-gis.mi.ingv.it>), (see Figure 4b).

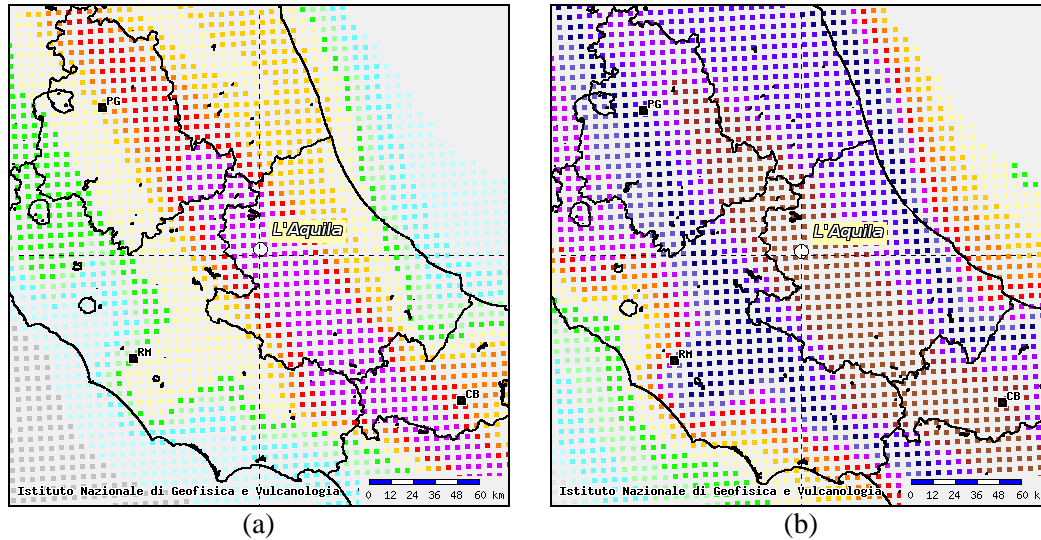


Figure 4. Hazard map of 10% probability of exceedance in 50 years for L'Aquila zone: PGA (a) and spectral acceleration at 0.4 seconds (b)

From these maps it may be recognized that PGA for L'Aquila is in the range (0.25-0.275)g while $S_a(T=0.4)$ is in the range (0.50-0.60)g. PGA and $S_a(T=0.4)$ ranges suggested by official Italian hazard data can be compared with the corresponding data recorded at the four stations closest to the epicentre.

Maximum PGA was 0.63g in the EW component of station AQV and minimum PGA was 0.34g in the NS component of station AQK, while $S_a(T=0.4)$ ranged from the minimum 0.32g of the EW component of station AQA to the maximum 1.36g of the EW component of station AQV. Considering possible soil amplification, not included in Figure 4 maps, it may still be noted that registered values exceed the expected value for a return period (T_R) of 475 years. Registered signals and earthquake characteristics can provide further information for the critical analysis of damage observed on RC buildings.

3.1 Spectral considerations

Seismic demand defined by previous codes can be considered equal to the lowest seismic capacity of structures designed according to them. The seismic demand of old

codes can be easily compared with the current code seismic demand and actual seismic demand registered during the 2009 L'Aquila event.

Eurocode 8 ([CEN 2003](#)) and the Italian Code ([DM 14/1/2008](#)) provide a Newmark-Hall functional expression for the elastic spectrum. By means of official Italian hazard data, given the specific geographic coordinates and ground type, the elastic spectrum may be defined for the considered site. Material, reinforced concrete in this study, and ductility class are necessary to define the behaviour factor q (see section 2); the latter is employed to pass from the elastic spectrum to the design spectrum.

Both Eurocode 8 and the Italian code provide two ductility classes depending on the hysteretic dissipation capacity. Both classes correspond to buildings designed, dimensioned and detailed in accordance with specific earthquake-resistant provisions, enabling the structure to develop stable mechanisms associated with large dissipation of hysteretic energy under repeated reversed loading, without suffering brittle failures.

Design spectrum defined by the Italian Code ([DM 14/1/2008](#)) for the Life Safety Limit State can be compared with the 1996 Ultimate Limit State spectral shape; the former was evaluated for both ductility classes, namely high ductility class (CD "A") and low ductility class (CD "B"), assuming a behaviour factor q determined for new design RC frame structures ([Figure 5a](#)).

Another interesting comparison can be made between the 1996 inelastic spectral shape and design response spectra evaluated for existing RC structures considering extreme values of q factor (1.5-3.0) according to the current Italian seismic code ([Circ. M.LL.PP. 617, 2009](#)), see [Figure 5b](#). Indeed, both Eurocode 8 and the Italian Code provide different values of behaviour factor q for new design and existing buildings, presuming that the latter cannot be characterized by properly high hysteretic dissipation capacity.

According to the Ultimate Limit State spectrum adopted in Italy between 1975 and 1996, a constant value of 0.105g was assumed for spectral ordinates between 0 and 0.8 seconds. This value for design inelastic acceleration can be reasonably considered representative of the minimum base shear coefficient of the great majority of L'Aquila RC buildings, if properly designed according to codes until 2003 and given ISTAT data ([Figure 2a](#)).

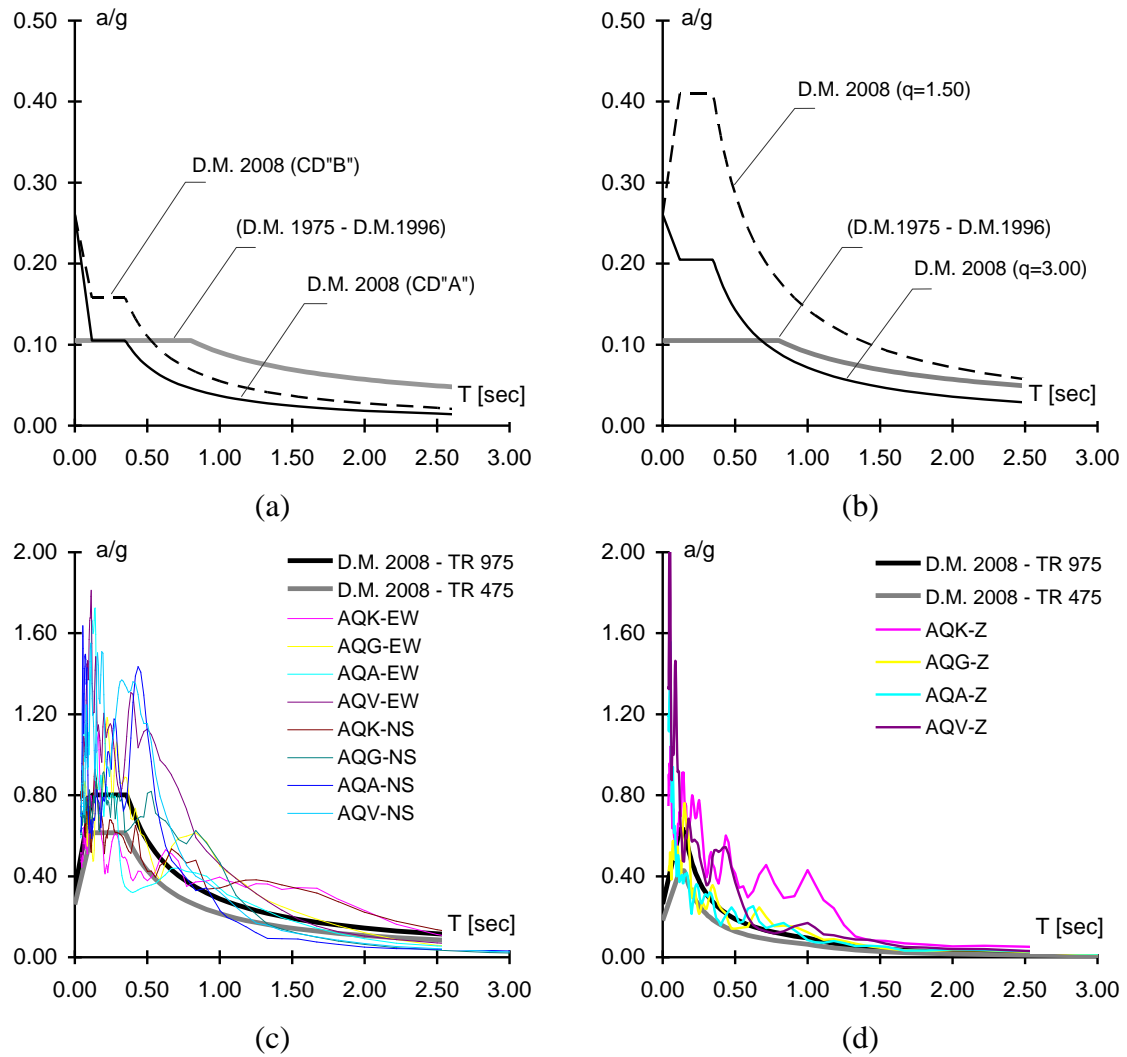


Figure 5. Inelastic old code spectra compared with current code spectra for new design structures (a) and existing structures (b). Current elastic code spectra compared with recorded signal spectra for horizontal (c) and vertical component (d)

Comparing constant acceleration branches of the various spectra shown in Figure 5a it may be noted that a building designed in CD “A”, and regular in plan and elevation, so characterized by a q factor of 5.85, according to the current Italian Code, is designed for the same seismic demand as in the old codes. However, it has to be considered that previous codes provided neither design rules nor detailed structural prescriptions able to ensure global and local ductility required by the current code, which allows adoption of a q factor of 5.85.

Generally, existing structures are unable to show a highly ductile behaviour and it is not possible to ensure that the structure develops stable mechanisms associated with large dissipation of hysteretic energy. This is why actual codes considerably limit q in such

cases, allowing it in the range (1.5-3.0). The choice of a value in this range should be made according to regularity criteria and the employment of material properties.

Hence, a proper comparison has to be carried out between the current seismic demand spectra provided for existing structures and the former code spectrum (Figure 5b). The latter comparison leads to a ratio of at least 2 between the current seismic demand and the old seismic demand.

Design procedures and details according to new generation codes, such as Eurocode, (BS EN 1990, 2002), for the Ultimate Limit State, considering the second reliability class (RC2), according to the same Eurocode definition (BS EN 1990, 2002), lead to an annual failure probability of at least $1 \cdot 10^{-6}$.

A ratio of current to old code demands, as calculated above, of at least 2, does not mean Life Safety limits are exceeded for all buildings; on the other hand, the percentage of building failure over the whole population, in this case, would definitely be higher than $1 \cdot 10^{-6}$.

If the elastic demand spectra of the registered signals are compared with the current code's elastic demand spectra, determined for different return periods (T_R 475 and 975 years) on ground type A, it may be noted that the spectra of the registered signals exceed code demand in most of the frequency range considered. Figure 5c compares elastic spectra determined from horizontal components of the registered signals in stations AQK, AQG, AQA and AQV, while Figure 5d compares vertical components of the same signals with the vertical code spectra.

In Section 2.2 it was emphasized that damage limitation limit state prescriptions and verifications essentially aim to avoid or reduce infill damage, and most notably that this kind of prescription was first introduced into the Italian code only in 1996 and better detailed and completed in 2003.

Hence, according to ISTAT data, it is reasonable to assume that most of the RC buildings in L'Aquila were constructed without any deformability control or verification (Figure 2a). On the other hand it should be emphasized that, even if a design procedure according to 1996 or, better, according to the 2008 code had been employed, involving Damage Limitation verification, the strong PGA characterizing the L'Aquila event would nonetheless have produced widespread damage to non-structural elements such as infills in many buildings in the area.

In the following a comparison between Damage Limitation spectra is proposed. The DM 1996 Damage Limitation spectrum can be extrapolated from the definition of elastic displacement demand (Eqn. 4). If a negligible influence of non-seismic loading is assumed ($\eta_p \approx 0$) and coefficient λ is assumed equal to 1, the displacement demand in verification of the limit state method is given by Eqn. 5. The acceleration spectrum employed to calculate η_d was that proposed in Figure 5a with a constant acceleration equal to 0.105g. Hence, if the Ultimate Limit State spectrum according to the 1996 code is multiplied by (2/1.5) and ϵ , β and I are assumed equal to 1, the expression in Eqn. 6 can be extrapolated and the damage limitation spectrum according to the 1996 code is obtained. This spectrum can be easily compared with Damage Limitation spectra of 2003 and current Italian seismic codes.

$$\eta_t = (2 \cdot \eta_d) / 1.5 \quad (5)$$

$$\begin{aligned} a/g &= 0.14 & \text{for } T \leq 0.8 \text{ sec} \\ a/g &= 0.14 \cdot 0.862/T^{2/3} & \text{for } T > 0.8 \text{ sec} \end{aligned} \quad (6)$$

Figure 6 shows Damage Limitation spectra according to different Italian codes, released in 1996, 2003 and 2008. Values corresponding to the constant acceleration part of the spectra are 0.14 g, 0.25 g and 0.242 g respectively.

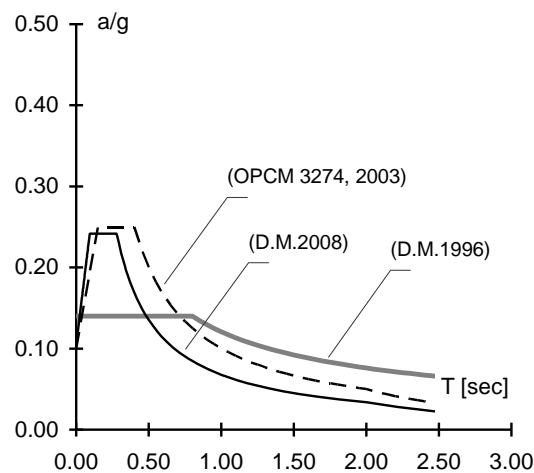


Figure 6. Comparison of damage limitation spectra: 1996, 2003 and 2008 codes.

As explained in Section 2.2, deformation capacity assumed by the 1996 code and later codes (2003 and 2008) strictly differ. As an example, given the same constraint condition between infill panels and the RC frame, the ratio between assumed capacities in the 1996 code and in later codes is $1/2.5$; conversely, the ratio between respective demands is $1.7 \div 1.8$. Based on this simple comparison, it may be concluded that the 1996 Damage Limitation restrictions were stricter.

4. STRUCTURAL DAMAGE

In this section the main structural damage to RC structures after the L'Aquila earthquake is presented. Photographic documentation ([Verderame et al. 2009](#)) was produced on the days immediately following the 6th April 2009 mainshock. Generally speaking, damage to structural elements is not so frequent and it seldom involves the whole structural system.

The main structural damage that involved RC columns can be easily recognized as failure caused by mechanisms that capacity design rules tend to avoid or at least to limit. During an earthquake, columns are characterized by high flexural and shear demand; maximum flexural demand combined with axial force produced by gravity loads and seismic loads are located at the end of the element; in these zones (critical regions) rotational ductility demand concentrates. Therefore, it is necessary to give an adequate rotational capacity and to avoid buckling of compressed longitudinal reinforcements.

Modern seismic codes, such as Eurocode 8 ([CEN 2003](#)), provide prescriptions to increase rotational capacity of the section: the upper limit on the longitudinal reinforcement percentage leads to a higher ultimate curvature of the section; proper hoop spacing and cross-tie presence give, due to a more efficient confinement action on concrete, an additional increase in section curvature capacity; finally, proper spacing between hoops avoids buckling in longitudinal reinforcement, or at least fixes an acceptable upper bound limit for which this phenomenon occurs.

However, the prescriptions and structural details presented above are typical of modern design concepts that first appeared in Italy in 1997. It is therefore possible to find RC columns with longitudinal reinforcement percentage exceeding 4% limit or hoops closed with 90° hooks, or with insufficiently thick spacing (15-20 cm).

Figure 7a presents a corner column of an RC building in the historical centre of L'Aquila, probably erected between 1950 and 1960. Damage occurred at the bottom end section of the element. The presence of smooth bars and a small hoop diameter (6 mm), closed with 90° hooks, can be observed, but the most significant detail is the absence of any transversal reinforcement in the first 30–40cm of the element immediately adjacent to the beam-column joint region. Figure 7b shows a circular column belonging to a building in the residential zone of Pettino, built in the 1980s: typical damage due to axial force and bending moment is recognizable; the concrete cover was crushed due to high compression strains and longitudinal bar buckling. In this case hoop spacing is, once again, not thick enough, as in the case of Figure 7a, but probably in this case the column was designed in accordance with code prescriptions at the time of construction.



(a)



(b)

Figure 7. Column with smooth bars and poor transversal reinforcement (a); damage to a column due to axial force and bending moment (b)

High shear demand can produce brittle failures with an outstanding reduction in column dissipative capacity. In order to prevent brittle failures, shear demand has to be determined according to flexural capacity of the element; applying to shear demand an amplifying coefficient to allow for variability in steel properties (CEN 2003) can prevent brittle failure occurrence. These prescriptions have been laid down since 2003 in the Italian code; no control of the failure mechanism used to be applied before this

code was released. All the above considerations can be confirmed by brittle failure of the columns reported in [Figure 8](#).

With regard to the rectangular column in [Figure 8a](#), whose section is $30 \times 100 \text{ cm}^2$, belonging to a 1980s' building, shear failure is evident, involving the top end section. Transversal reinforcement has hoop spacing of approximately 15-20cm, and is definitely under-designed with respect to column section size, that is, with respect to the inertia of the section, thus leading to premature shear failure of the element. The brittle failure mechanism is highlighted by the crushing of the concrete within the reinforcement and the complete opening of the third and fourth hoops from the top end of the element.

[Figure 8b](#) shows shear failure of a 30cm-diameter circular column; in this case it is possible to recognize insufficient hoop spacing, which leads to the typical diagonal cracking characteristic of shear failure mechanisms and longitudinal bar buckling in the column.

In order to stress the non-secondary role played by column – infill interaction in determining brittle failures in the structural elements, [Figure 8c](#) shows the damage that columns present in these kinds of situation. It is possible to recognize the brittle failure in the column due to the local interaction with the concrete infill partially covering the bay frame, reaching 1/3 of the total column height. Partial infilling that effectively interacts with the column reduces the slenderness of the element and consequently produces a higher shear demand that exceeds column shear capacity. This kind of phenomenon involves all the columns interacting with the concrete partial infilling.

[Figure 8d](#) gives an example of a basement that is partially below the ground level. According to common building practice, basement levels are characterized by walls, often realized in concrete, aimed at a retaining function of the adjacent embankment; concrete wall height is limited with respect to column height to allow the fitting of windows. This structural solution leads to a strict reduction in column slenderness with a consequent increase in shear demand; moreover, decreasing shear span can modify the shear span ratio of the element up to a squat column behaviour. This situation is of no secondary importance since the shear resistance mechanism of a squat column differs with respect to the typical behaviour of a slender element. Differences between shear capacity formulations proposed in Eurocode 8 ([CEN 2005](#)) for existing buildings for

slender and squat columns testify to the difference between shear failure mechanisms. Hence, if local interaction between the column and concrete wall is not allowed for, premature brittle failure due to excessive concrete compression can often occur.



Figure 8. Shear failure of: (a) rectangular and (b) circular (b) columns, (c) column adjacent to partial infilling panels, (d) squat column adjacent to basement level concrete walls

Columns belonging to staircases can easily show brittle failures as well. Most common staircase types generally possess discontinuity elements in the regular RC frame scheme composed by beams and columns. Indeed, on one side a staircase consists of inclined

axis elements (beam or slab); on the other, squat columns are created by the intersection of inclined axis elements with the column.

Staircase elements lend a considerable lateral stiffness to the whole structural system, first due to axial stiffness of the inclined elements and secondly to higher lateral stiffness of squat columns. These contributions are easily appreciable via linear analyses. On the other hand, staircase elements can represent a weak point because they possess higher shear demand that can lead to brittle failure mechanisms.

Figure 9 shows a staircase composed by inclined beams; the squat column in the corner has typical shear failure. Poor transversal reinforcement, both in terms of hoop spacing and diameter, can be recognized.



Figure 9. Shear failure in squat columns of the staircase

Shear failures were found in reinforced concrete as well. As an example, in **Figure 10** two reinforced concrete walls, respectively with two different shape factors, are shown; damage consists of a spread diagonal cracking. Low longitudinal and transversal reinforcement percentages can be recognized, especially compared with minimum values prescribed by design codes based on capacity design approach.

Beam-column joints can completely change the structural behaviour of the whole building and their failure should necessarily be avoided in a proper seismic design approach: such elements possess brittle failure mechanisms. In geometrically very small beam-column joints, demand coming from beams and columns is concentrated and both the concrete panel and longitudinal bars are subjected to high gradients of shear and flexural demand.

Joint failure mechanisms are mainly governed by shear and bond mechanisms; force distribution, which allows shear and moment transfer, produces diagonal cracking and hence joint failure due to diagonal compression in the concrete is quite likely to occur, thus producing a reduction in strength and stiffness in the connection.



Figure 10. Failure in reinforced concrete walls

Generally speaking, joint design is limited by concrete compressive stress; the diagonal stress induced by the elements meeting in the joint cannot exceed concrete compressive stress. In order to keep structural continuity when concrete cracking occurs, a proper transversal reinforcement along the whole element should be provided. The presence of transversal reinforcement allows stresses to be transferred by means of a strut and tie mechanism, even if the cracking phase has passed in the concrete. The latter mechanism can be developed if longitudinal reinforcement, transversal reinforcement and concrete struts contribute to truss formation. If capacity design prescriptions are followed, preventing brittle failure in joints gives the chance to develop more ductile mechanisms in the other structural elements.

Specific design rules for beam-column joints appeared in the Italian design prescriptions only in 2003 ([OPCM 3274, 2003](#)). Indeed, in the explanatory document to the 1996 code ([Circ. M.LL.PP. 65, 1997](#)) the transversal reinforcement in the joints was simply required to be at least equal to the hoop spacing in the columns.

Damage from the 6th April 2009 earthquake clearly shows how hazardous failure of joints can be. [Figure 11a](#) shows an external beam-column joint, characterized by an extensive cracking in the joint panel. The absence of transversal reinforcement leads to local buckling of the longitudinal bars that consequently results in concrete cover spalling. Interestingly, the absence of proper transversal reinforcement in the joint also leads to a loss of anchorage in beam longitudinal reinforcement.

[Figure 11b](#) shows typical diagonal cracking failure in a concrete panel belonging to an external joint. Cracking begins at the intersection between the joint and upper column and ends at the intersection between the joint and lower column, producing the loss of monolithic connection. Absence of hoops, in this situation too, leads to buckling in the external longitudinal bars and involves lower column bars without transversal reinforcement in the first 30–40cm.

Another noteworthy aspect in RC damage after the L'Aquila earthquake is a peculiar loss of connection at the lower joint-column interface; this aspect is emphasized and becomes a critical issue when there is insufficient longitudinal and transverse reinforcement both within the joint and at the column end.

Generally speaking, the presence of a separation (previously present or otherwise) at the interface between the column and beam-column joint, if both elements – column and joint – are well designed, complying with modern seismic prescriptions, should not prevent the development of a ductile failure mechanism in the column. According to current code prescriptions, (i) a minimum amount of longitudinal reinforcement, evenly distributed around the periphery of the section, and (ii) an adequate, effectively anchored transversal reinforcement, both in the beam-column joint and at the column end, have to be adopted, thus avoiding brittle failure along the interface section between column and joint ([Paulay and Priestley, 1992](#)).

[Figure 11c](#) reports a brittle failure mechanism due to the lack of transverse and longitudinal reinforcement that led to loss of continuity at the intersection between the joint and lower column, being the probable final cause of the failure. [Figure 11d](#) shows a clear separation in concrete at the joint-column interface; in this case, due to a complete spalling of the concrete cover, the reinforcement in the element may be recognized and it may be noted that the first hoop in the column is partially open. The

lack of transverse reinforcement in the element made the separation at the joint-column interface critical.



Figure 11. Joint failure with evident (a) longitudinal bar buckling and (b) diagonal cracking failure in concrete joint panel; (c) (d) failure mechanisms at joint – column interface surfaces

5. NONSTRUCTURAL DAMAGE

As a general rule, infill failure mechanisms can be classified as: (i) horizontal sliding in the central zone of the infill panel, (ii) diagonal cracking due to tensile stress in the central zone of the infill panel, (iii) corner crushing in the direct contact application zone.

Figure 12 shows two building facades, in which infill panels are characterized by a diagonal cracking mechanism. In the first case (Figure 12a) it is worth noting how cracking diffusion involves infillings adjacent to window openings; in the second case damage is concentrated at the first storeys of the building (Figure 12b). In Figure 12b

diagonal cracking is more emphasized by the plaster layer because the external layer of the infill is made up of solid clay bricks that experienced limited damage.

Figure 12c shows a typical corner crushing mechanism. Out-of-plane failure of the infilling external layer allows the corner crushing mechanism of the internal layer to be observed; other evidence is the crack, which visibly involves the plaster but is also probably deeper, localized at the top of the column adjacent to the infill panel, as a consequence of column-infill local interaction.



Figure 12. Infill panel failures: diagonal cracking (a), (b) and corner crushing (c) mechanisms

The great majority of external infill panels consist of double layer infill panels; internal layers are generally made with clay bricks, connections between the two layers by the interposition of brick elements discretely, or lined up. The low efficacy of this system is emphasized in Figure 13a.

Furthermore, in most of the observed cases, internal infilling layers are restrained at the four corners of the RC frame while external layers are constrained only by the upper and the lower beam by means of a small pawl. This constructive solution leads to a decrease in the interaction mechanism between RC frame and external infill panel, for

both in-plane and out-of-plane seismic forces. The low efficacy of the restraint applied to the external panel, coupled with the ineffective or completely absent connections between the two layers, leads to damage limited in most cases to the external infill panel which can easily show an out-of-plane failure due to seismic action in both directions, as can be observed in [Figure 13b](#). Both neither local nor global interaction effects between infills and the RC structure are negligible. As was previously emphasized, local interaction between infill panel and adjacent columns can lead to (i) a reduction in the effective height of the column, an increase in shear demand and a consequent brittle failure of the column when the infill panel partially occupies the frame bay; (ii) shear concentration demand at the end of the column and consequent brittle failure when diagonal compression is applied by the panel on the RC element.



(a)



(b)

Figure 13. External infill panel failures without connection between layers (a) and with ineffective connection (b)

As a global phenomenon infill-structure interaction increases global stiffness of the complex system and consequently spectral acceleration demand. Besides, it can represent a source of irregularity in plan or elevation (e.g. *pilotis*) when the infill distribution is irregular.

Some particular cases of structural failure after the L'Aquila event and mainly caused by irregularities in plan or elevation are reported in [Figure 14](#). The first structure (see [Figure 14a](#)) was situated in the centre of L'Aquila (Via Porta Napoli); it was irregular in elevation and the second storey had an evident discontinuity in terms of infill distribution; in the left wing of the building there was a sort of porch. In terms of

building collapse, damage was concentrated on the second storey, leading to complete failure of the upper levels.



(a)



(b)

Figure 14. Soft storey mechanism examples in L'Aquila: (a) Via Porta Napoli, (b) Via Dante Alighieri (Pettino)

The structure proposed in [Figure 14b](#) was placed in the residential zone of Pettino (Via Dante Alighieri), close to L'Aquila. The building has an irregular shape in plan, similar to a T; infill distribution makes the structure irregular in elevation due to the presence of garage entrances. The building showed a soft-storey mechanism at the first storey that

can be explained by infill irregularities in elevation and by a local interaction between infills and adjacent columns, which probably led to a brittle failure of some columns at the first level, and consequently leading to the collapse of the whole building. An analytical study of this collapse can be found in [Verderame et al. \(2010\)](#).

CONCLUSIONS

RC buildings in L'Aquila area do not represent the main part of the whole building stock that mainly comprises masonry buildings no higher than three storeys. Thanks to a detailed review of seismic codes in force in past decades, an analysis was made of the weaknesses that can be found in existing buildings. Damage recorded on RC structures after the L'Aquila mainshock event was then analyzed in the light of previous considerations about construction practices.

Several general conclusions can be drawn about the performance of RC buildings:

- The main damage involved non-structural elements such as infill panels.
- Structural damage was found in columns, walls and beam-column joints, but most of the times it was caused by design procedures, consistently with the code prescriptions in force at the age of construction, not properly aimed at avoiding brittle failure mechanisms.
- Documented building collapses were essentially caused by irregularities in plan or elevation of the structural and non-structural complex.

As a side consideration, it should be emphasized that seismic demand associated to the mainshock event exceeds the seismic demand characteristic of the $T_R=475$ years event determined on stiff soil; the latter can be considered as the main benchmark since it represents the seismic demand of the Life Safety Limit State for an ordinary building when designed according to the seismic code currently in force in Italy.

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